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Technical Report

R 234

EVALUATION OF 40- BY 100-FOOT ARCH-RIB UTILITY BUILDING

3 May 1963

U. S. NAVAL CIVIL ENGINEERING LABORATORY

Port Hueneme, California

EVALUATION OF 40- BY 100-FOOT ARCH-RIB UTILITY BUILDING

Y-F015-99-.29

Type B

by

R. M. Webb

ABSTRACT

A 40- by 100-foot arch-rib building, prefabricated of 12-gage-steel ribs and 26-gage galvanized sheeting by Trim-Steel, Incorporated, was erected and subjected to the loading specified in the Uniform Military Requirements Criteria for Prefabricated Advanced Base Buildings. The results of the tests were as follows:

Test	Requirements	Results
Snow load	20 psf	36 psf
Wind load	70 mph	94 mph
Weathertightness	weathertight	satisfactory
Weight	—	19,425 lbs
Cubage	—	274 cu ft
Erection time	350	323 manhours

It is concluded that the building meets the minimum requirements of the criteria for Advanced Base Buildings.

CONTENTS

Figure 1. 40- by 100-foot Trim-Steel Building.

Figure 2. Interior of the Trim-Steel Building.

INTRODUCTION

The U. S. Naval Civil Engineering Laboratory was assigned the test and evaluation of a promising, commercially available, prefabricated metal building. Manufactured by Trim-Steel, Incorporated, of Spokane, Washington, this arch-rib utility building was studied and tested for suitability of packaging and crating, ease of erection, structural adequacy, weathertightness, and economy.

Building Description

The Trim-Steel Building (Figures 1 and 2), which is approximately semicircular in cross section, is 40 feet wide by 100 feet long. It is 18 feet high at the crown. The structure consists of arch ribs, purlins, sheeting, and endwall framing, and is similar to the "Quonset." The 12-gage-steel building ribs are 8 by 6 inches; their cross section is similar to a hat-shaped trapezoid. The ribs are spaced on 5-foot centers and are secured to the foundation by base brackets which are attached to anchor bolts embedded in the concrete footing (Figure 3).

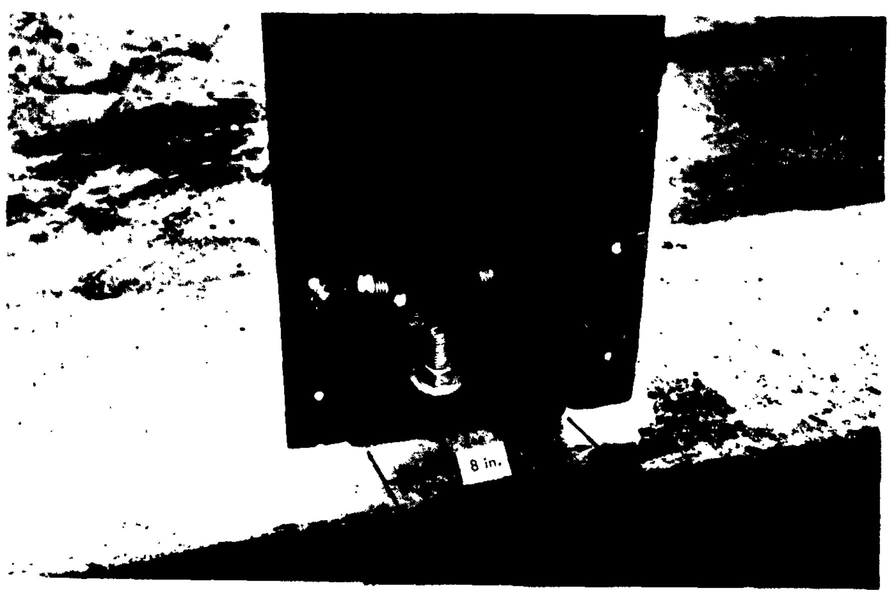


Figure 3. Details at springing of arch rib.

The purlins are 3-1/4-inch by 1-1/2-inch by 16-gage steel. They are hat-shaped and are spaced on 4-foot 10-inch centers about the circumference of the ribs. Each purlin is bolted to each rib with four self-tapping screws.

The sheeting is corrugated, galvanized, 26-gage steel which is precurved to the radius of the building. Individual sheets are 27-1/2 inches wide and are used in 64- and 125-inch lengths. The sheets are attached to the purlins with No. 14 cadmium-plated bolts spaced on 8-inch centers. The bolts have cadmium-plated washers to which neoprene washers are vulcanized.

The endwalls are framed with 4-inch by 3-inch by 12-gage angle ribs continuous over 4-3/4-inch by 2-inch by 14-gage channel studs. The studs are faced with 29-gage galvanized steel sheeting attached with 1-inch No. 14 bolts spaced on 8-inch centers. Lap joints in the sheeting are bolted on 12-inch centers. Each endwall provides a 14- by 16-foot double sliding door, 20 square feet of glazed area, two 2-foot by 3-foot 9-inch fixed louvers, and a nominal 3- by 7-foot walk-in door.

To insure weatherproofing, flashing is placed over the junctions between the roof and endwalls, and mastic is applied to all joints in the sheeting and flashing. Because the bottom of sheets adjacent to the foundation rests slightly above the base of an offset (blockout or lip) in the concrete, a continuous fillet of mastic is placed to provide a seal between the bottom of the sheeting and the base of the offset.

With exception of the ribs and purlins, the prefabricated parts of the building are galvanized or cadmium-plated to protect against corrosion. The ribs and purlins have a baked-on enamel finish. Sidewall walk-in doors and windows, translucent natural-light panels, and screened louvers and roof ventilators are available but were not evaluated.

Test Facility and Instrumentation

The structural facility for test of the building components was fabricated of standard Navy pontoons and structural steel beams. In this facility an instrumented section of the building was subjected to simulated snow and wind loading (Figure 4).

Simulated loadings were applied by means of hydraulic cylinders actuated by a 12-channel hydraulic console (Figure 5).

The cylinders were attached to 1/4-inch-diameter wire ropes extending between the cylinders and the structure. The wire ropes incorporated strain-gaged tension links. Other instrumentation consisted of 48 electrical-resistance-type SR-4 strain gages and 20 mechanical clockface-dial deflection gages.

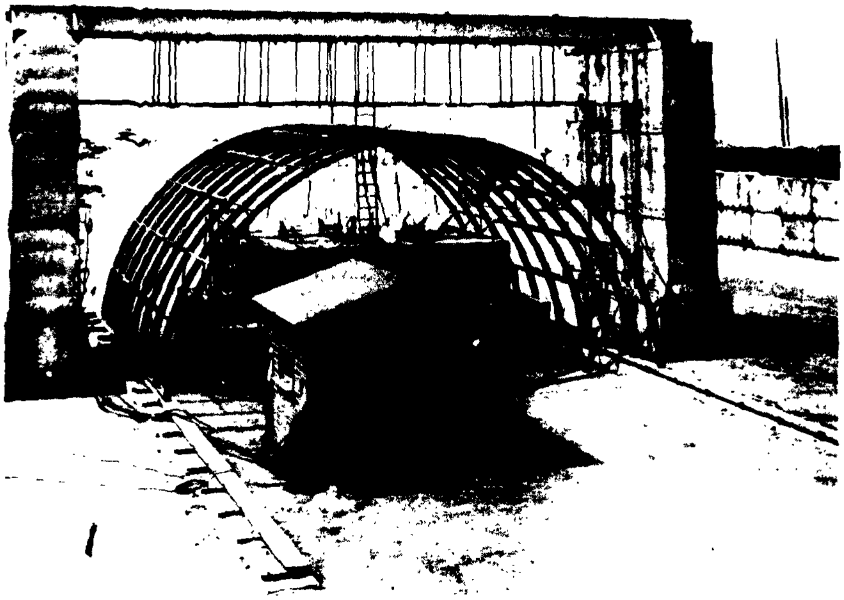


Figure 4. Test facility and test section.

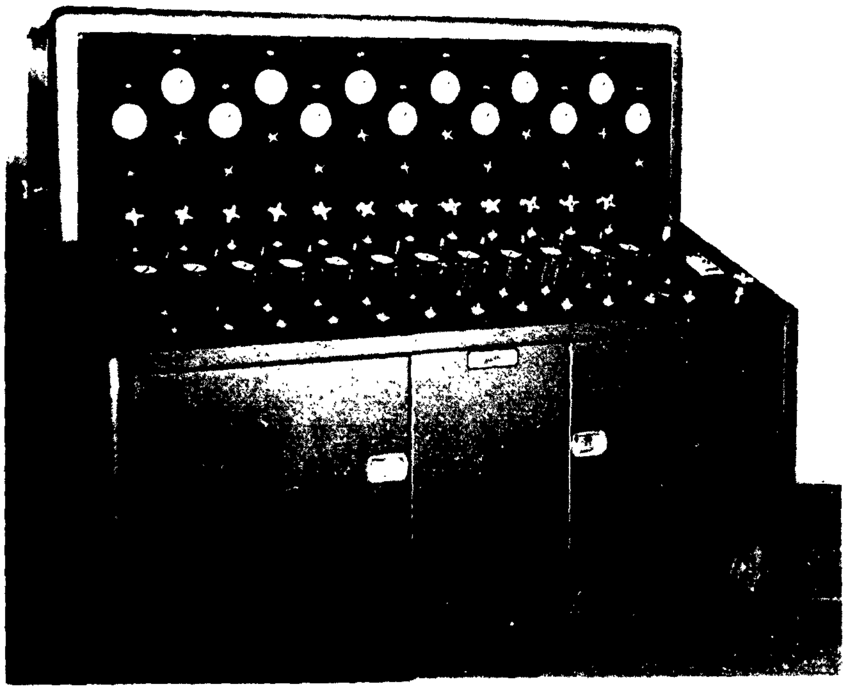


Figure 5. Hydraulic console for applying loads.

Points on the periphery of the structure to which hydraulic jacks, deflection gages, and strain gages were attached are shown in Figures 6a and 6b. With the exception of deflection gages D-18 and D-19, all deflection gages and strain gages were attached to the center rib of the test section. Gage D-17 was located midway between gages D-2 and D-3 because it was initially estimated that the deflection at this point would be greater than that at point D-2. The tension links T-1 through T-20 (which were incorporated in the wire ropes) were placed as shown in the plan and elevation views of Figure 6b. The loadings were of greater intensity over the area bounded by hydraulic channels Ch-5 and Ch-7, and turnbuckles were incorporated with the tension links in the wire rope assemblages for these channels so that the thrust from each jack could be individually adjusted. Thus, friction within the jacks could be accounted for by adjusting the turnbuckles, and the deflection of the ribs adjacent to the instrumented center rib could be adjusted as required to minimize the load distribution effect of the purlins. Thirteen channels were required to simulate snow loading, but the hydraulic console contained only 12 channels. Therefore, the tangential load components for channels Ch-3 and Ch-9 were applied by using six jacks in channel Ch-1, as shown in Figure 6b.

TESTS AND SIGNIFICANT DATA

Packaging

When the building components were received from the manufacturer, the packaging was inspected for adequacy, and the packages were weighed and measured. Then the packaged building was placed under canvas tarps in open storage for a period of ten months. During this period, the building materials were subject to corrosion. Significant data from these tests are: (1) the weight and cubage of the packaged building were 19,525 pounds and 274 cubic feet, respectively, and (2) initial stages of corrosion occurred in the purlins.

The dimensions and weight of each package are tabulated in Table VIII of the Appendix.

Erection

The building was erected to determine the manhour requirements and the adaptability of the building for multiple erection. The erection was made by a crew of 10 men, which consisted of two 5-man working parties. Except as noted elsewhere, each party was composed of one forklift truck operator, two ironworkers (who placed all sheets and tightened all bolts), one ironworker helper (who assisted in aligning sheets on the building), and one general laborer (who placed joint caulking compound on sheets, etc.).

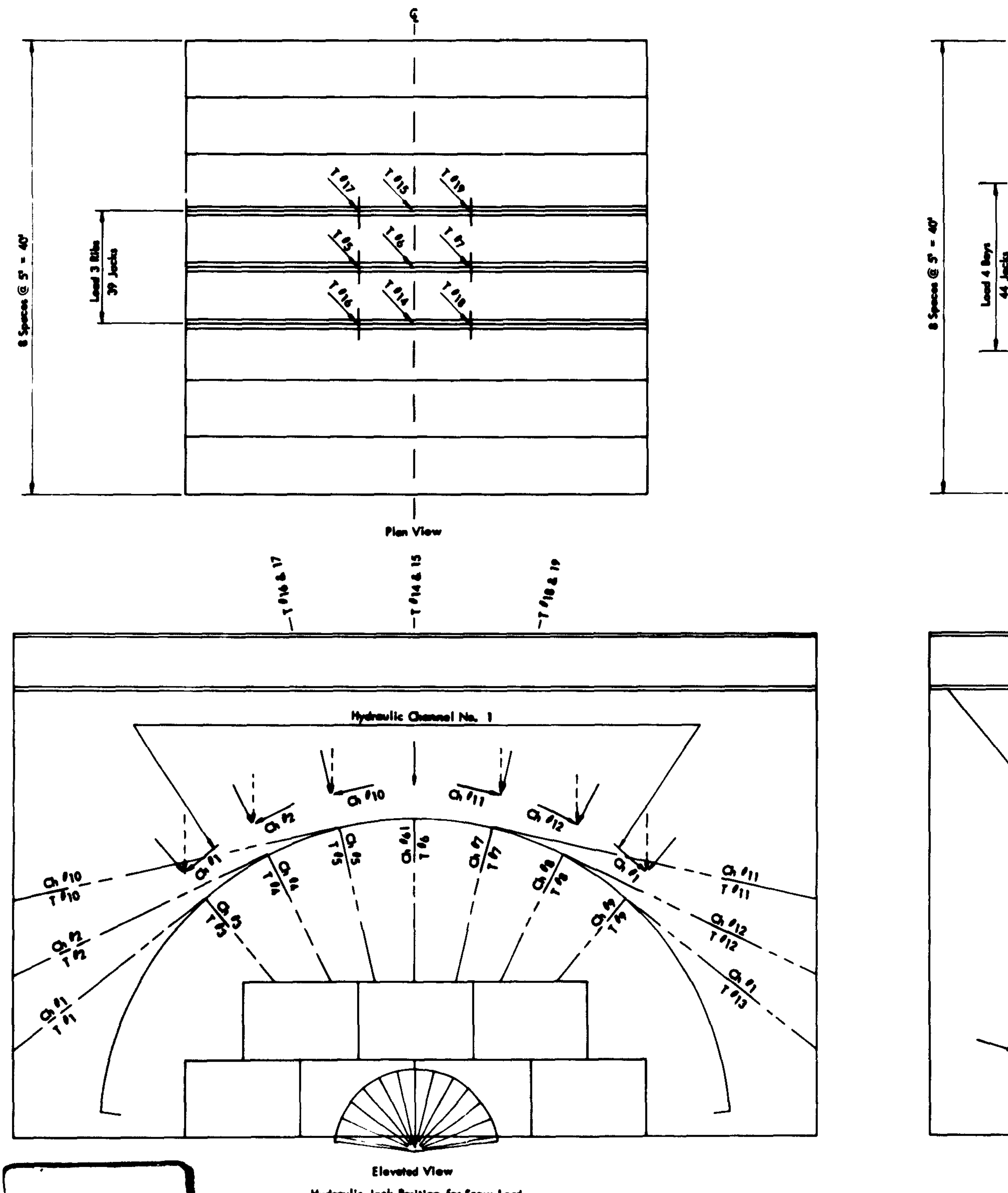
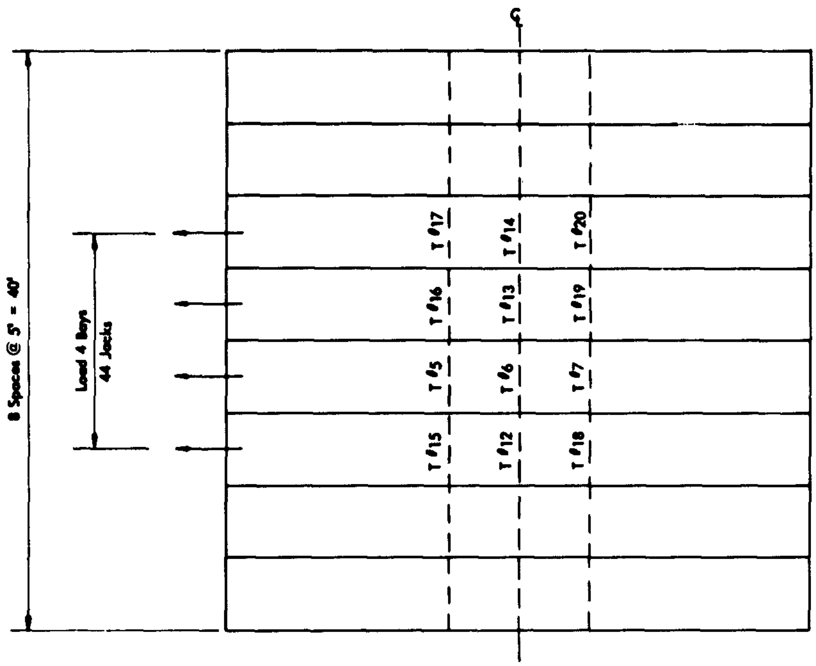
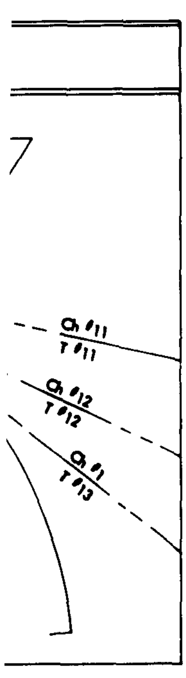
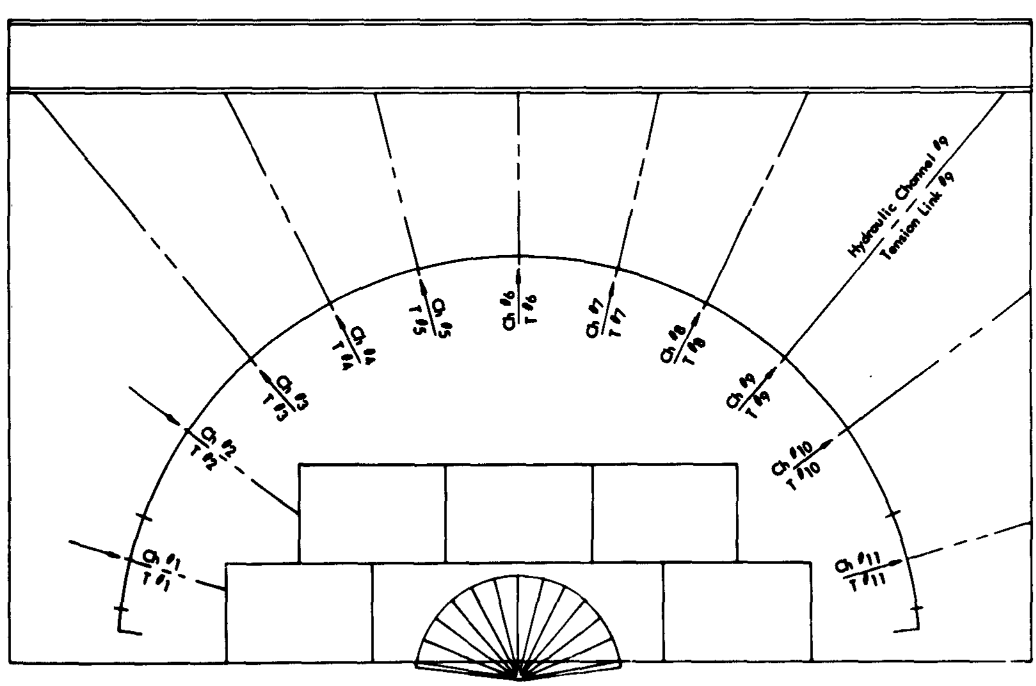


Figure 6b. Diagram showing points to which hydraulic tack



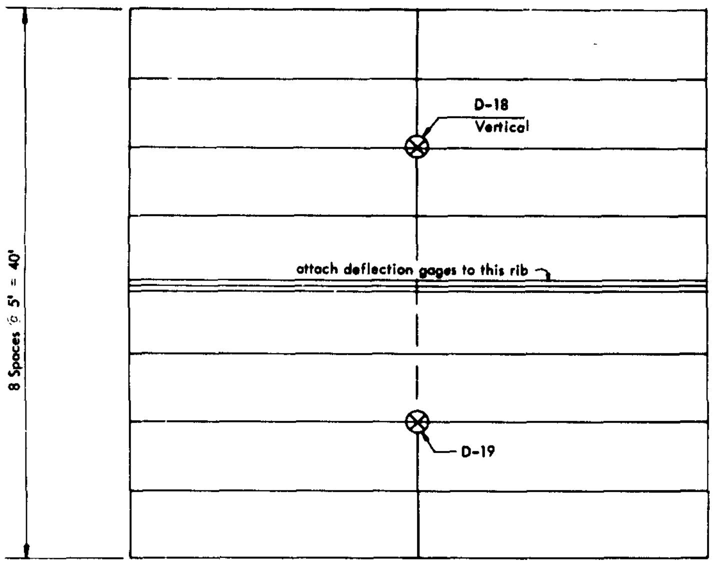
Plan View



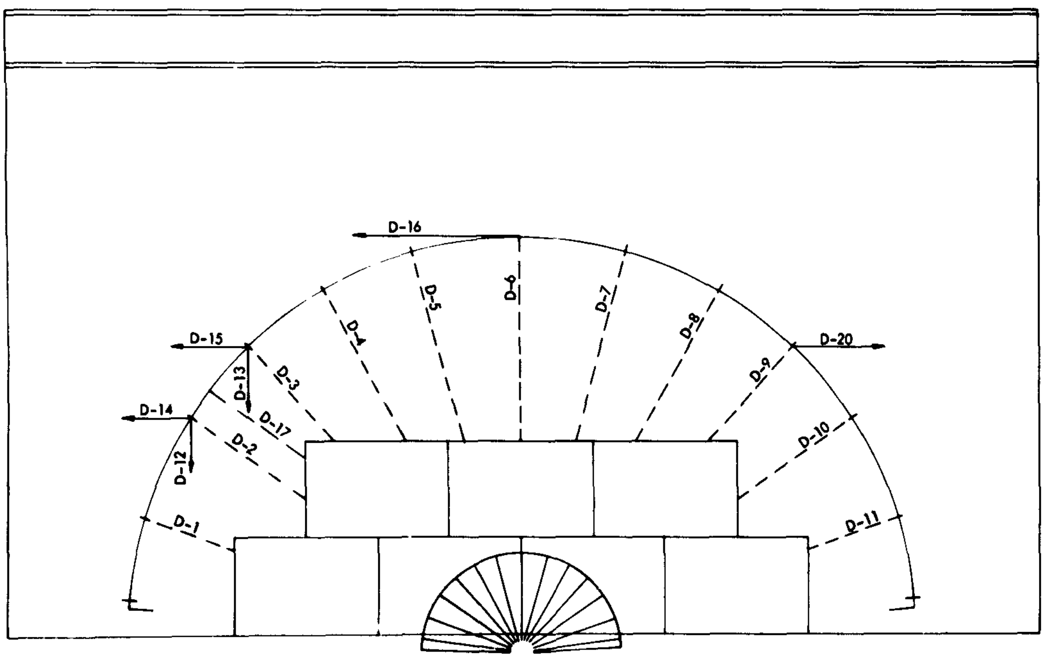


Elevated View
Hydraulic Jack Position for Wind Load

owing points to which hydraulic tacks were applied.

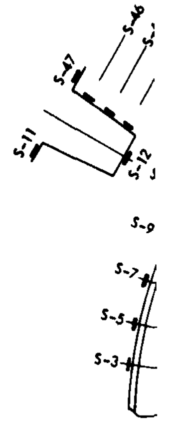


Plan View



Elevated View

Deflection Gage Position



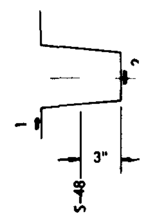
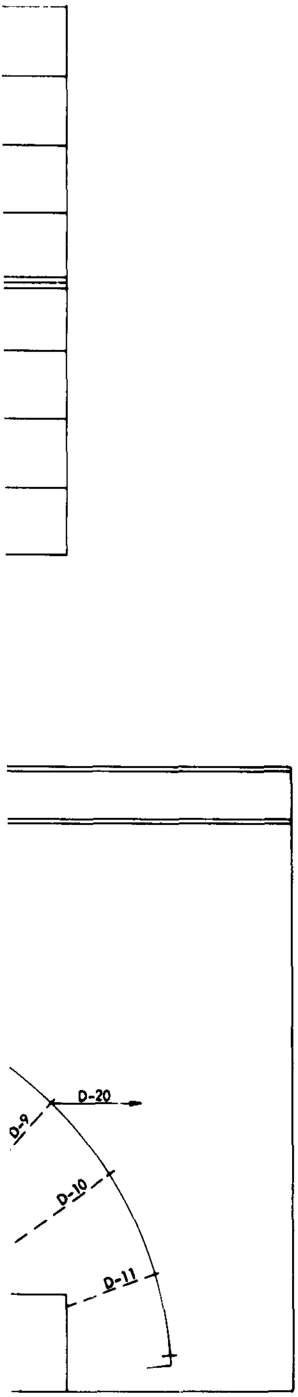
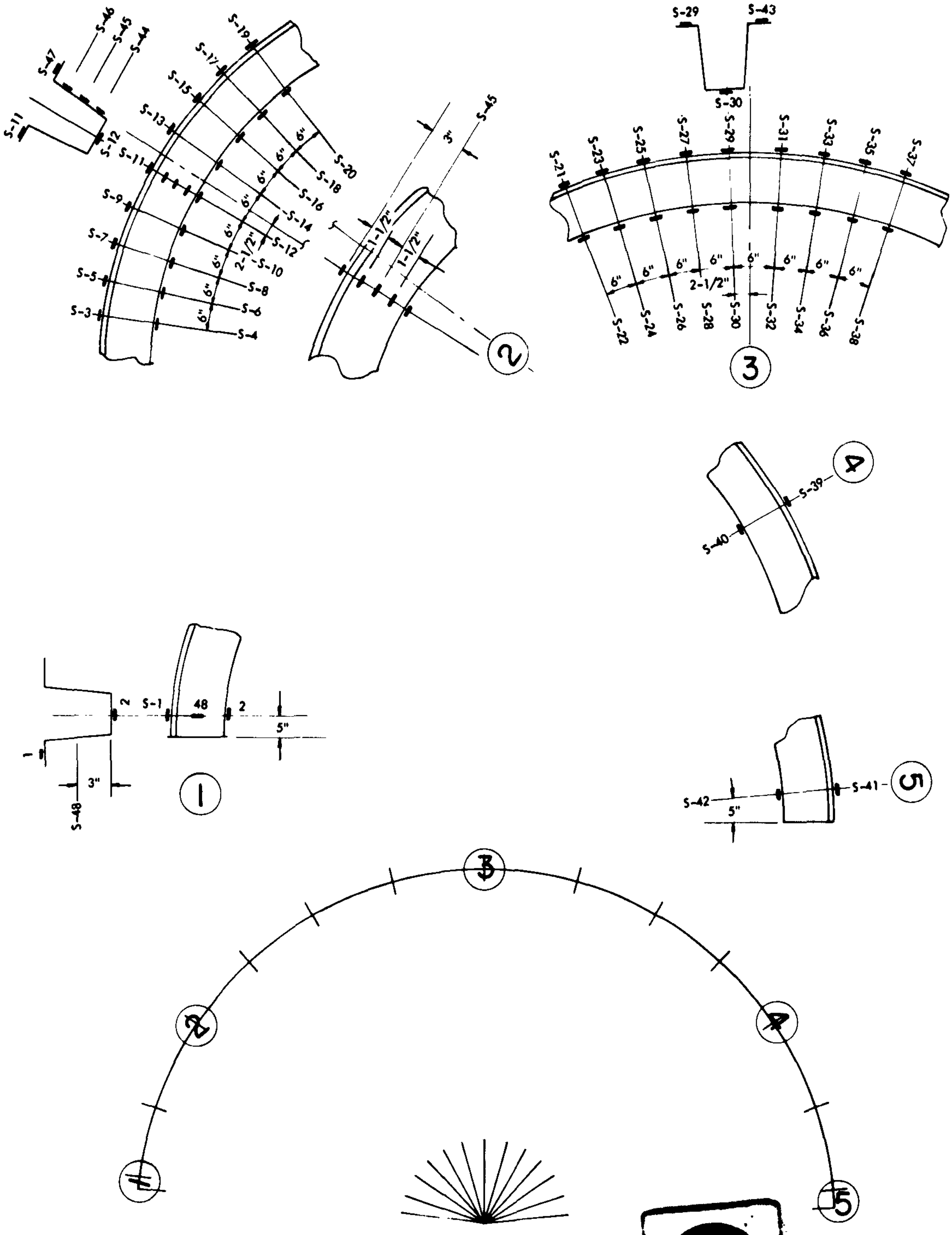


Figure 6a. Diagram showing points to which deflection





Strain Gage Placement Diagram

ıgram showing points to which deflection gages and strain gages were applied.

2

During erection of the ribs and purlins, a crane and two forklift trucks were used, but by substituting an operator for a laborer the crew was held to 10 men.

Tools used during erection are shown in the illustrations which follow. A complete listing of the tools is given in Table IX of the Appendix; these tools were not furnished by the building manufacturer.

Erection of the building components was divided into four parts: (1) ribs and purlins, (2) endwall framing, (3) roof sheeting, and (4) building flashing. The procedure for erection of these parts follows, with significant data.

Ribs and Purlins. First, all ribs were assembled on the ground. Then, in sequence and by use of a crane and a manila rope choker attached to the crown of the rib, each rib was hoisted to the foundation. As the rib was hoisted, a workman at each springing guided the rib into place on the foundation. All ribs were attached to the foundation by anchor bolts (Figure 3). As the ribs were placed, purlins were placed and bolted to the ribs. Thus, the frame was stabilized laterally.

Scaffolds for the workmen were provided by steel platforms mounted on the forks of forklift trucks.

Endwall Framing. The rib, studs, girts, cover sheets, and door leaves were installed by a crew of 5 men. Bolt holes were prepunched for erection of the structural members, but field-drilled holes were required for installation of the sheeting (Figure 7). The cover sheets were placed horizontally, and the laps between adjacent sheets were weatherproofed with mastic. Self-tapping metal screws were used to fasten the sheets to the structural frame and to stitch the lap joints between adjacent sheets. The screws were tightened with an electric-driven screwdriver, "Scrugun." The 3- by 7-foot walk-in doors and the 2-foot 9-inch by 3-foot 9-inch windows were installed without difficulty.

Roof Sheeting. The sheeting was placed, caulked, and fastened as shown in Figures 8 and 9. Lap joints in the sheeting were caulked by use of hand-operated mastic guns and stitched with self-tapping metal screws tightened by "Scruguns."

Although not purchased or tested with the building, 13 skylight panels were placed symmetrically in the roof, as shown in Figures 1 and 2.

Building Flashing. Steel flashing was applied to the junctions between the endwalls and roof and to the openings around the windows and walk-in doors in the endwalls. Adequate prefabricated flashing was provided, but it had to be field-cut to fit.

Figure 7. Installation of building sheeting.

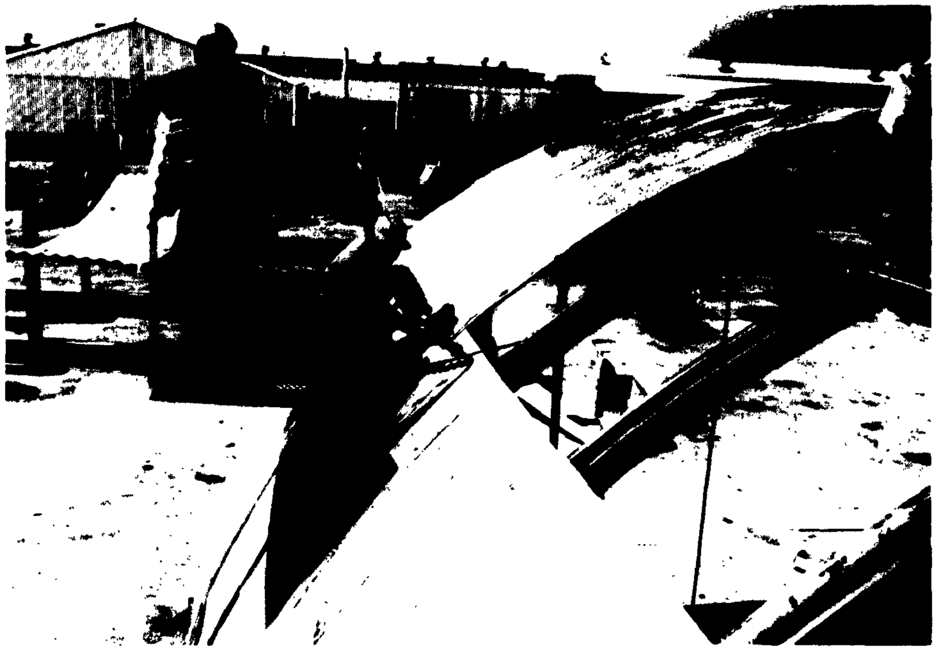


Figure 8. Installation of roof sheeting.

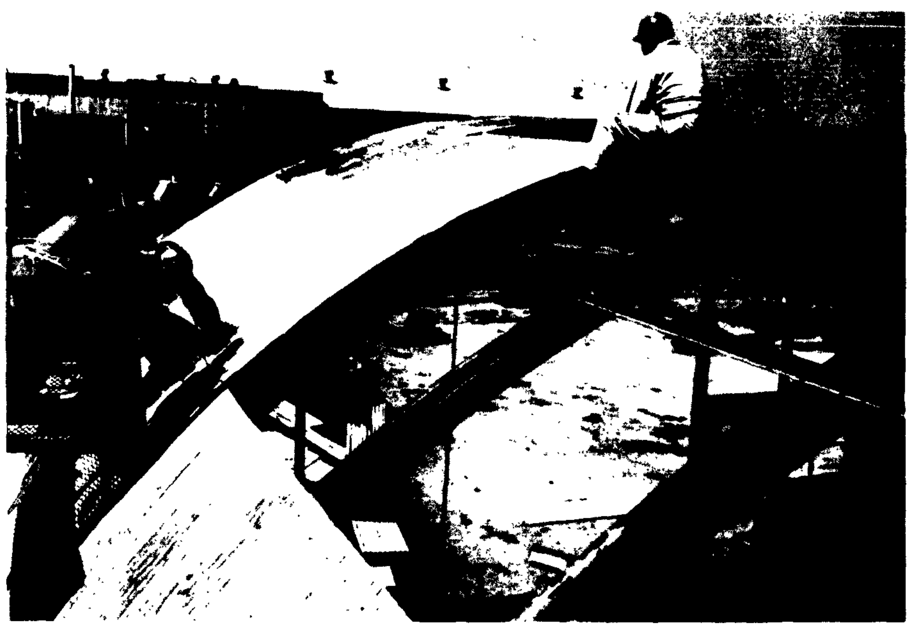


Figure 9. Stitching roof sheeting.

The manhour requirements for erection are given in Table I.

Table I. Manhour Requirements for Erection by 10-Man Crew

Item		Manhours
Ribs and purlins		68
Endwall framing		93
Roof sheeting		157
Building flashing		5
	Total	323

Structure

Structural tests were divided into three major parts: (1) arch tests, (2) endwall tests, and (3) a sheeting test.

Arch Tests. An unsheeted 40-foot section of the building (nine ribs spaced 5 feet center to center) was used for test of an arch rib to withstand a 20-psf snow load or a 70-mph wind load.[1] To simulate the response to load of ribs in the center portion of a 100-foot-long building, it was necessary to load only the center and two adjacent ribs because the effect of endwall restraint on these ribs would be negligible. Thus, the other six ribs provided longitudinal stability for the test section.

Radial and tangential components of loads were applied to the three center ribs by means of the hydraulic jack system. For snow load, concentrated loads were applied directly on the ribs; for wind load, concentrated loads were placed at the center points in the middle of the span of the purlins, which span continuously across the ribs (Figure 6). To stiffen the purlins for transmitting loads to the ribs, 2-inch by 6-inch by 5-foot-long planks were bolted to the purlins. The loads were applied in 20 percent increments from zero to 180 percent of the load specified in the test criteria.

Snow Load Test: Snow load was assumed to be of constant intensity, w, over the central 40-degree portion of the arch and to vary cosinusoidally from w to zero over the remaining portions, which terminate where the slope of the arch is equal to 45 degrees.[1] The loading was applied in increments of 4 psf from zero to 36 psf; thus, the maximum load was equal to 180 percent of the 20-psf design load. Because the structure was to be tested for wind load, and as buckling failure could develop below the yield stress of the rib material, a load factor of 1.8 was considered to be adequate for this test, even though the maximum stress was less than 70 percent of the yield stress.

The load-deflection diagram for the test rib is given in Figure 10. In this diagram the curves give the relation between the unit load and the radial deflection.

The equations of stress for three points on the arch rib (A, B, and C, Figure 11) are given in Table II. These linear equations were derived by the method of least squares, and they give the best straight-line fit of the data for strain gages S-3, -4, -12, -29, -30, and -47. In deriving the equations, the modulus of elasticity was assumed to be 30×10^6 psi because the stress-strain diagram given (Figure 22 in the Appendix) was obtained from test of only a single specimen of the rib.

Table II. Arch-Rib Stress

Point 1/	S (top fibers)	S (bottom fibers)
A	+(830w - 1,586)	-(1,176w - 9,433)
B	+(534w - 5,209)	-(1,175w - 9,120)
C	-(662w - 5,209)	+(829w - 916)

1/ See Figure 11.
\+ Tension.
\- Compression.

Note: In Table II, S is the unit stress in psi and w is the unit load in psf; the coefficients and constants are expressed to the nearest whole number.

For the three locations of Table II, the percentages of compression stress due to bending and axial thrust are given in Table III.

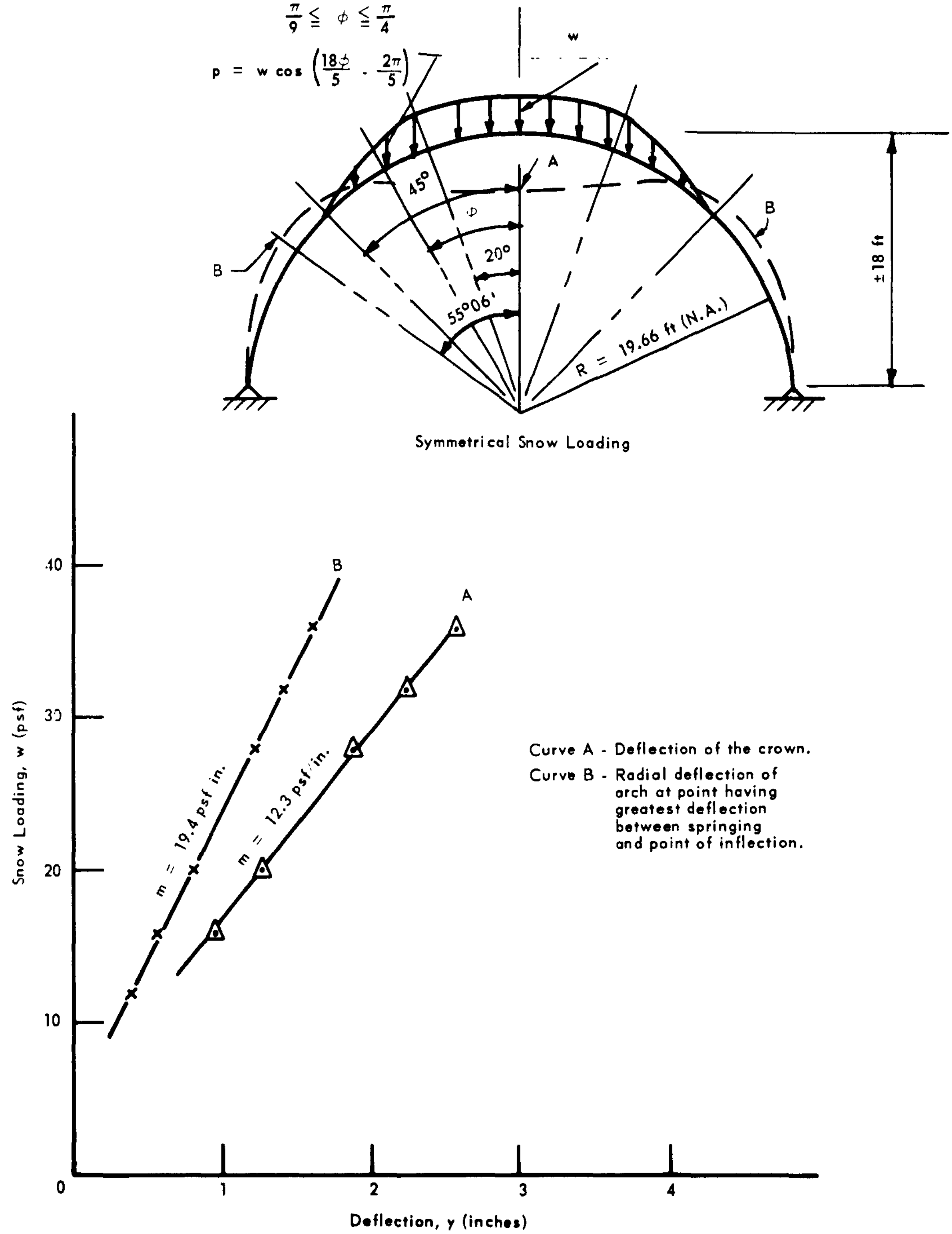
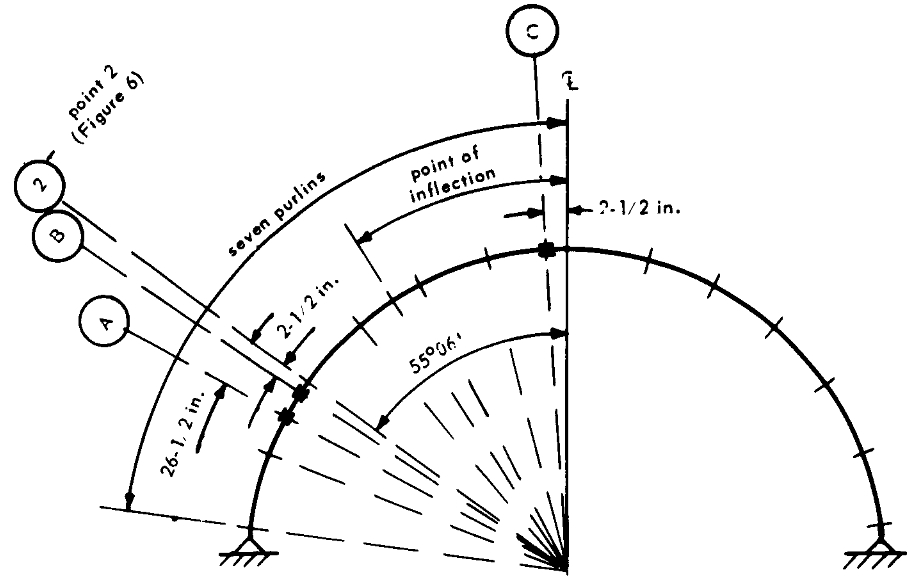


Figure 10. Load-deflection diagram for snow loading.



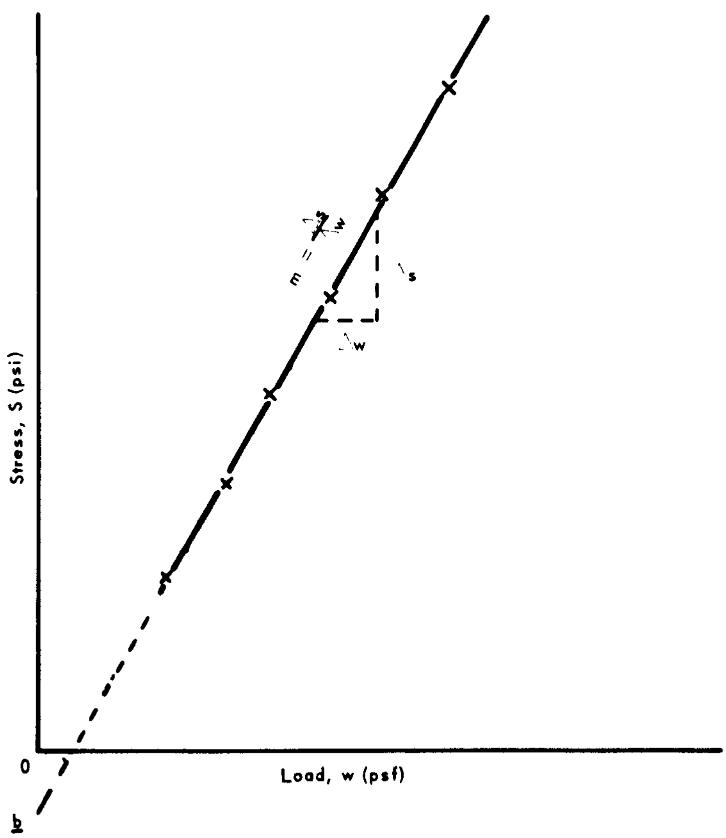


Figure 11. Stress-load curve for snow load.

Table III. Arch-Rib Stress Components

Point[1]	Bending Stress[2] (percent)	Axial Stress (percent)
A	97	3
B	92	8
C	94	6

1/ See Figure 11 and Table II.
2/ Percent of the total compression stress.

Note: The components of stress in Table III are derived from the strain data in Table X of the Appendix.

Wind Load Test: The wind loading pattern is shown in Figure 12.[1] The loading was applied in 20 percent increments from zero to 180 percent of the pressure resulting from a 70-mph wind. Therefore, the maximum pressure was equivalent to a 94-mph wind. A maximum strain of 915 microinches occurred at point A (Figure 12) and was equal to 85 percent of the maximum which occurred in the previous test for snow load.

The load-deflection diagram for the test rib is given in Figure 12. A stress-pressure curve for the point of maximum stress (point A, Figure 12) is given in Figure 13. Supplementary data for the wind load test are tabulated in Tables XII and XIII of the Appendix.

Endwall Tests. Tests of the endwall were designed to include the effect of wind on the endwalls when the building was subjected to snow load. Three tests were made: (1) composite framing test, (2) door test, and (3) door jamb test. Details of the individual tests follow.

Composite Framing Test: This test was made in two parts. For the first part, the crown portion of the arch was loaded with sand to simulate snow as shown in Figure 14. The sand load extended 15 feet in from the endwall and was 21 psf of horizontal projection of the loaded portion. The stress and deflection of various framing members were obtained with SR-4 strain gages and mechanical deflection gages, as indicated in Figure 15. The stress and deflection at the center of the door header span were 1,500 psi and 0.22 inches, respectively. The deflection data for members subjected to bending in this part of the test are given in Table IV. The axial thrust transmitted to the door jamb was 370 pounds.

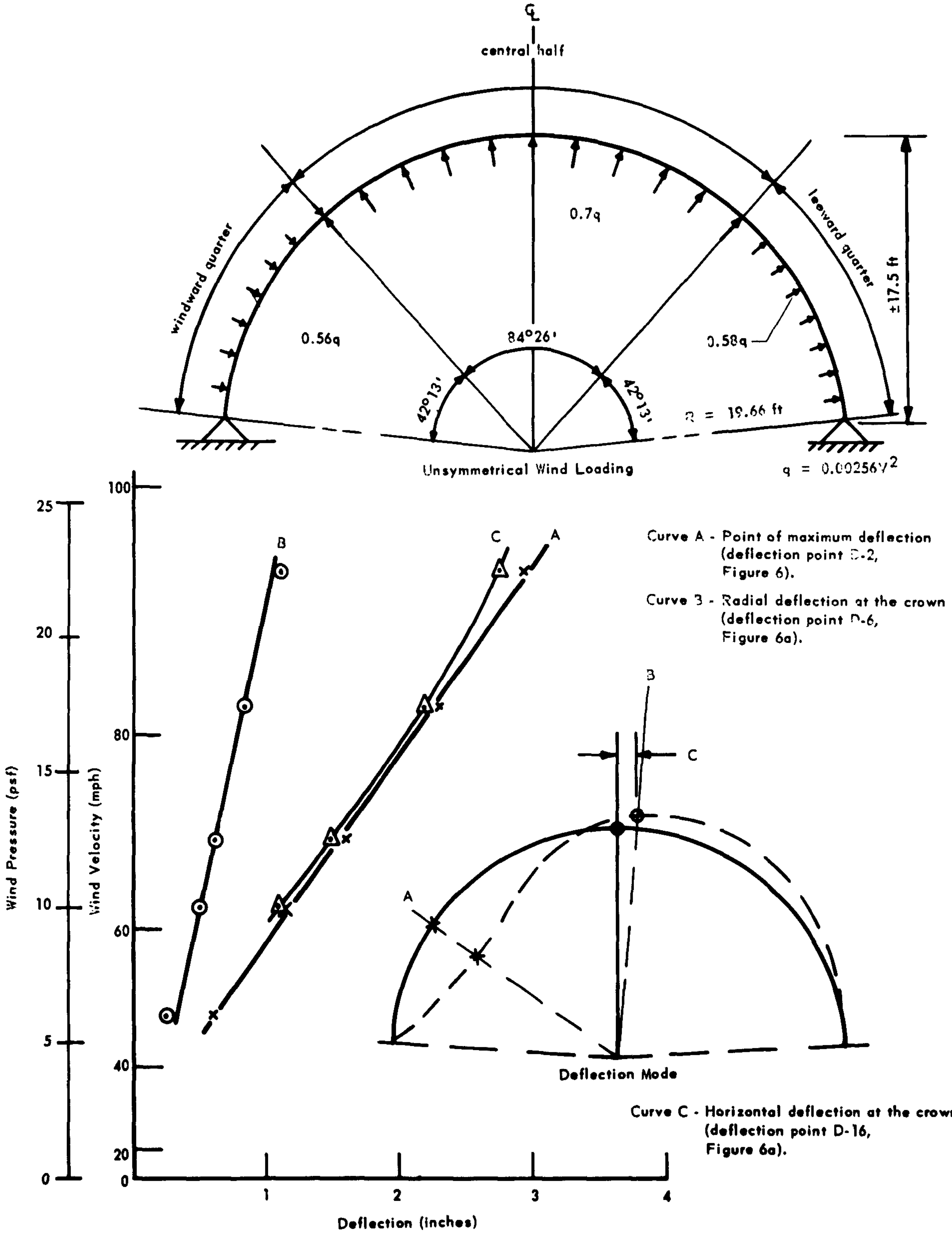


Figure 12. Load-deflection diagram for wind loading.

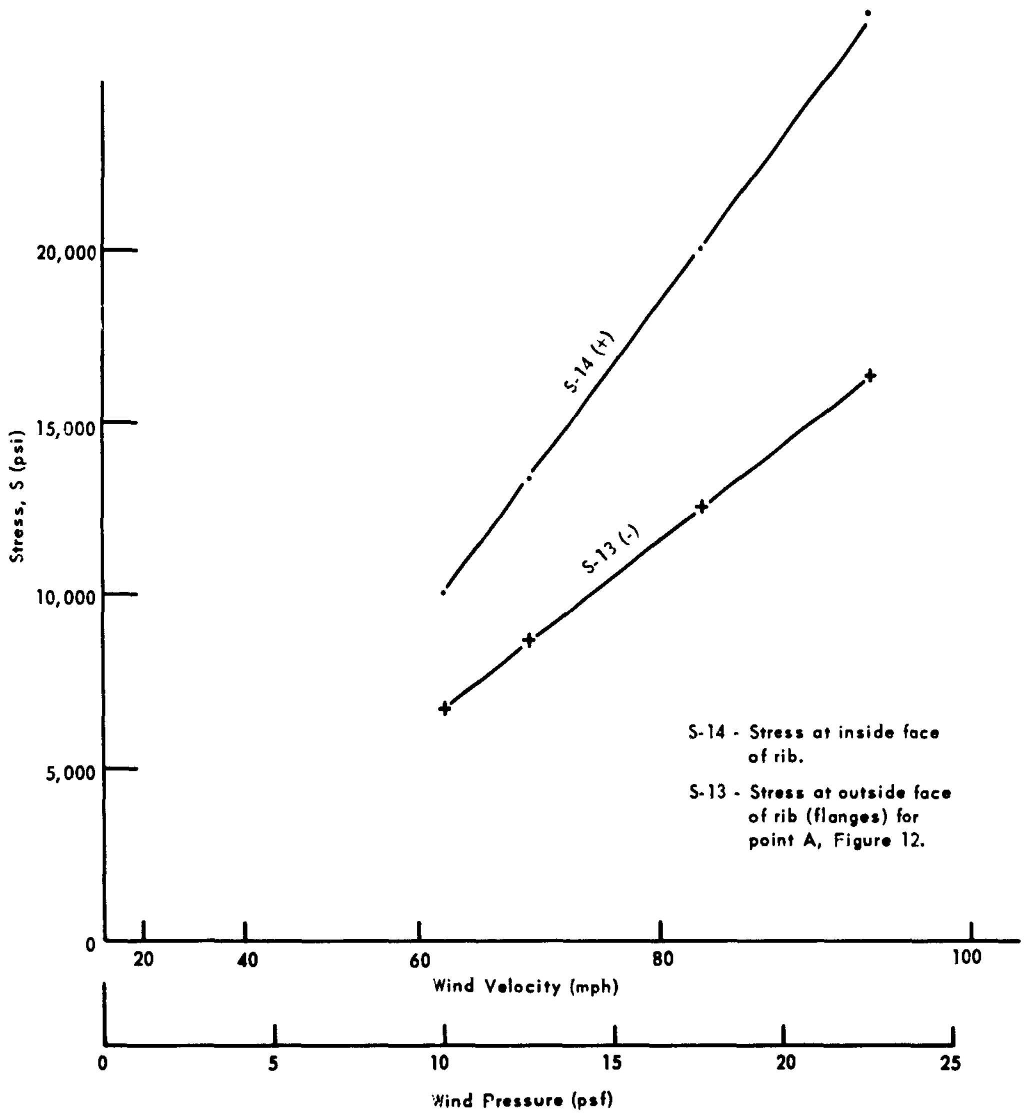


Figure 13. Stress-pressure diagram for wind loading.

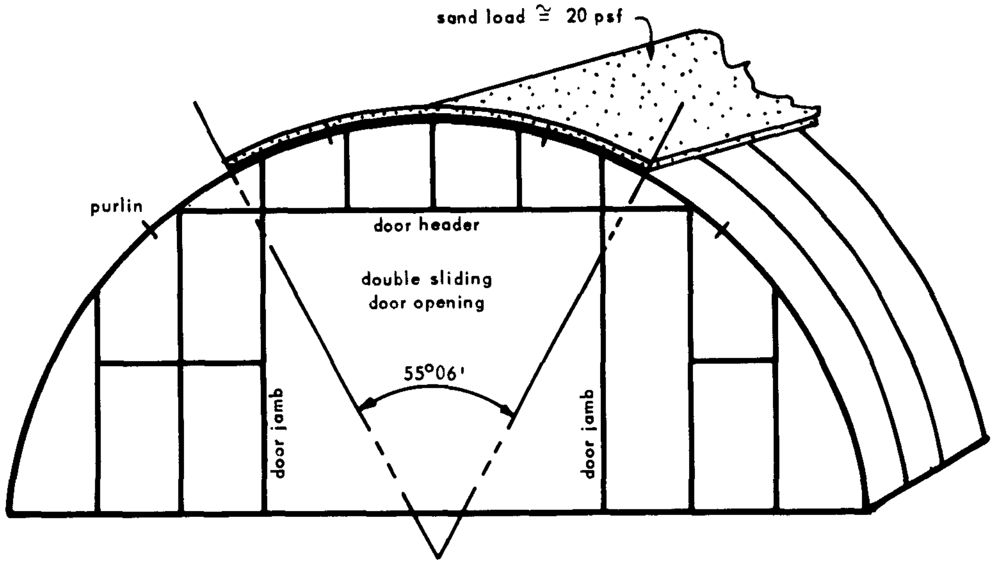


Figure 14. Sketch showing location of sand loading.

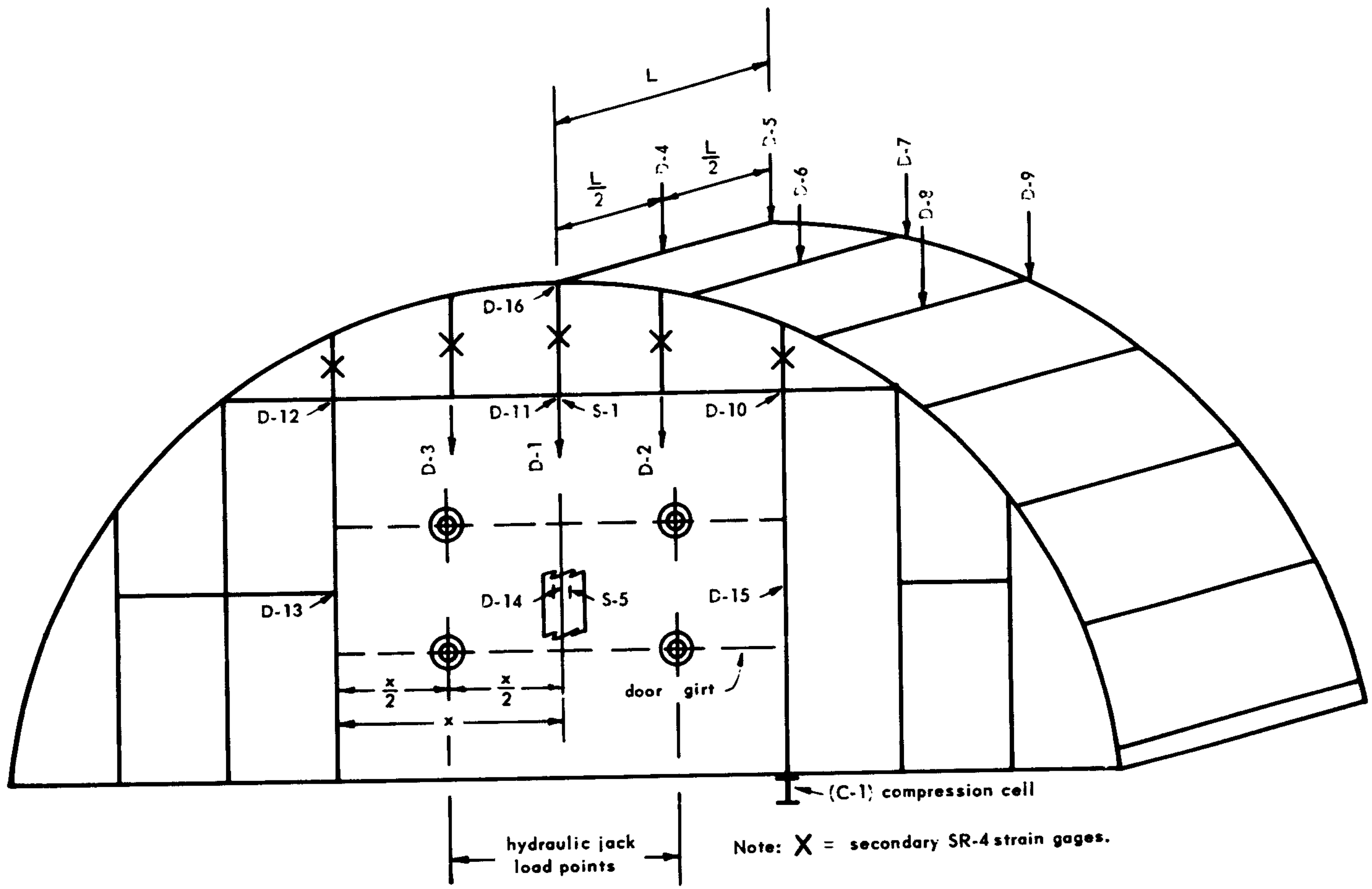


Figure 15. A sketch of endwall framing showing load and instrumentation stations.

Table IV. Deflection Data for Axial Load on the Endwall Framing

Gage No.	Deflection 1/ (inches)
D-1	0.22
D-2	0.18
D-3	0.18
D-4	0.81
D-5	0.95
D-6	0.58
D-7	0.72
D-8	0.52
D-9	0.78

1/ Deflection due to sand loading of 21 psf (Figure 14).

For the second part of the test, the sand load was left on the structure. Then to simulate wind, hydraulic jack loads were applied to the double sliding doors as indicated in Figure 15. Thus, the door jambs were subjected to axial loading and transverse bending, and the door header was subjected to biaxial bending. The stress at the centerline of the door (gage S-5, Figure 15) varied linearly, from zero to 26,370 psi, as the unit wind load was varied from zero to 12 psf. Deflection data for this part of the test is given in Table V.

Door Test: One 8- by 14-foot section of the double sliding door was tested as a flat plate with three edges simply supported and the remaining 14-foot edge elastically supported at the junction of the girts and leaf (Figure 16). The elastic supports were provided by springs inserted in wire rope assemblages which incorporated SR-4 strain-gaged tension links, and thereby, the distribution of load to the girts was determined.

The door was subjected to a uniformly distributed sand load of 11.82 psf. The weight of the sheeting was 0.98 psf; therefore, the total unit load on the door framing was 12.80 psf. The deflection of the leaf was 0.81 inches and was obtained by measuring the displacement of the springs (Figure 16). For a total load of 1,328 pounds the reactions of the girts were 154 pounds and 143 pounds from tension links Nos. 1 and 2, respectively.

Table V. Deflection Data for Axial and Lateral Loading on Endwall Framing

Unit Wind Load (psf)	Deflection[1/] (inches)						
	D-10	D-11	D-12	D-13	D-14	D-15	D-16
5.3	0.35	0.78	0.40	0.35	1.30	0.50	0.18
6.7	0.48	1.00	0.50	0.52	1.60	0.70	0.25
8.0	0.61	1.25	0.62	0.69	1.92	0.90	0.32
9.3	0.72	1.48	0.72	0.79	2.02	1.00	0.38
10.7	0.92	1.81	0.92	0.96	2.52	1.18	0.42
12.0	1.11	2.12	1.10	1.12	2.88	1.33	0.50

1/ All deflections directed inward; axial load due to sand (Figure 14) and lateral load due to wind (Figure 15).

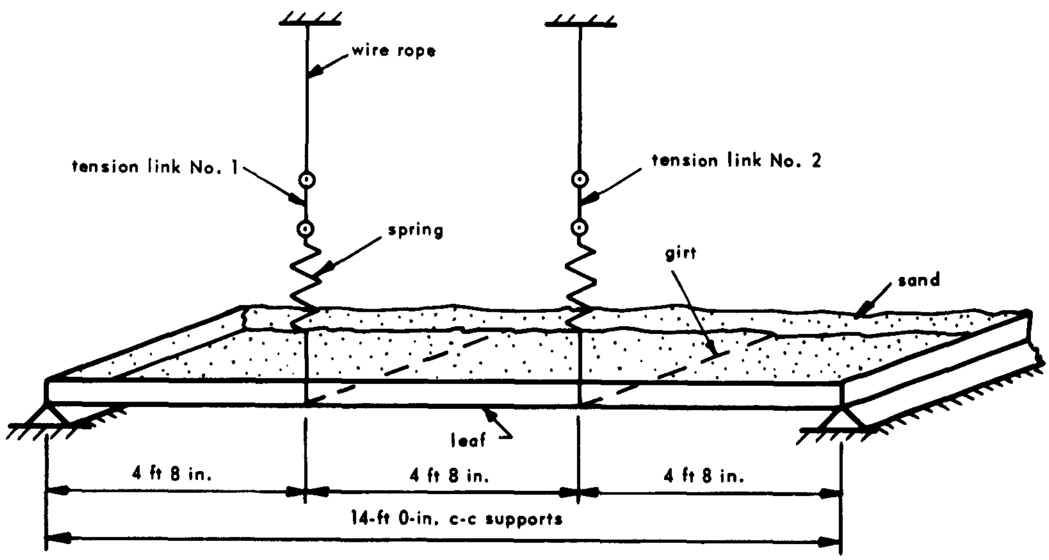


Figure 16. Partial isometric sketch of test door.

Door Jamb Test: A door jamb was tested as a simply supported beam column (Figure 17). The jamb was subjected to loadings simulating those transmitted to it in test of the composite frame. The data for this test is given in Table VI.

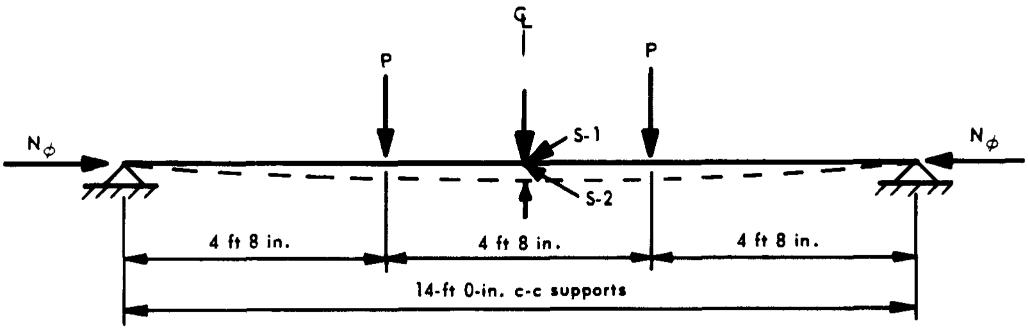


Figure 17. Sketch of door jamb showing axial loads, N_φ; transverse loads, P; deflection, y; and SR-4 strain gages, S-1 and S-2.

Table VI. Data for Test of Door Jamb[1/]

N (pounds)	P (pounds)	y (inches)	Stress[2/] (psi)	
			S-1	S-2
370	0	0.01	–	–
370	100	0.52	-3,000	+1,650
370	200	0.93	-9,600	+3,900
370	300	1.50	-16,800	+5,100

1/ See Figure 17.
2/ Based on E = 30×10^6 psi.

Sheeting Test. A 51-inch width of sheeting was tested to determine its stiffness as an arch. To perform this test, the purlin and sheeting at the crown of the frame used in the composite endwall test were removed; then, the sheeting was partially replaced by the test arch (Figure 18). To prevent displacement at springing, the purlin supports for the arch were braced horizontally and vertically. Then, the arch was subjected to a concentrated load in pounds, P, placed at the crown.

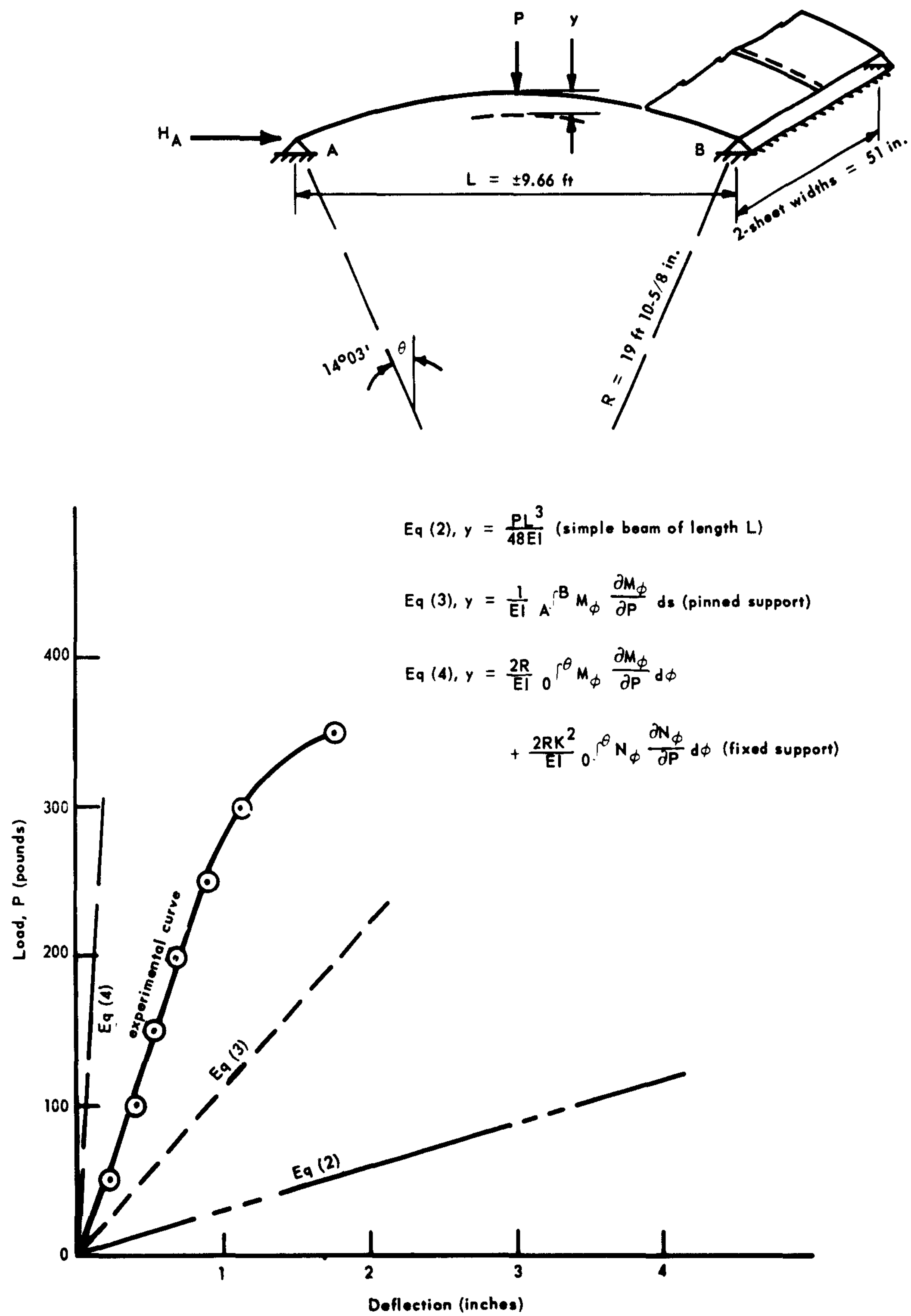


Figure 18. Load-deflection diagram for building sheeting.

The load-deflection diagram for the arch and three theoretical curves derived by use of Castigliano's theorem[2] are given in Figure 18. The theoretical curves are based on E equal to 30,000,000 psi and I equal to 0.00746 $inch^4$ per foot of width.[3]

Weathertightness

The building was evaluated for weathertightness by applying a simulated 2-inch-per-hour rainfall accompanied by a 35-mph wind.[1] The rainfall was applied through a fire hose equipped with a spray nozzle, and the water supply was metered. The intensity of rainfall was based upon the horizontal projection of the building area covered by the spray. The wind velocity was created by a portable turbine blower calibrated for control of the velocity pressure produced at various distances from the discharge orifice.

Leaks occurred at 16 spots where caulking was inadequate, but these leaks were easily stopped by application of additional mastic.

RESULTS AND DISCUSSION

Packaging

The volume of the packaged building was 274 cubic feet, and it weighed 19,525 pounds. The packaging material weighed 477 pounds, and the weight of metal in the building was 19,048 pounds. The packaging material consisted mainly of banding (Figure 19). Computations show that by regrouping the ribs, purlins, and sheeting, the cubage could be reduced to approximately 250 cubic feet.

The packaged building was placed under canvas tarps in open storage for a period of ten months. During this period, initial stages of corrosion occurred on the purlins as shown in Figure 20.

Erection

For comparison, the cubage, weight, and erection time for the Trim-Steel Building and similar buildings are given in Table VII. Except for the shorter erection time for the Wonder Building, the Trim-Steel Building compares favorably with buildings previously evaluated by the Laboratory.

The 10-man crew consisting of two 5-man teams was efficient in erecting the building.

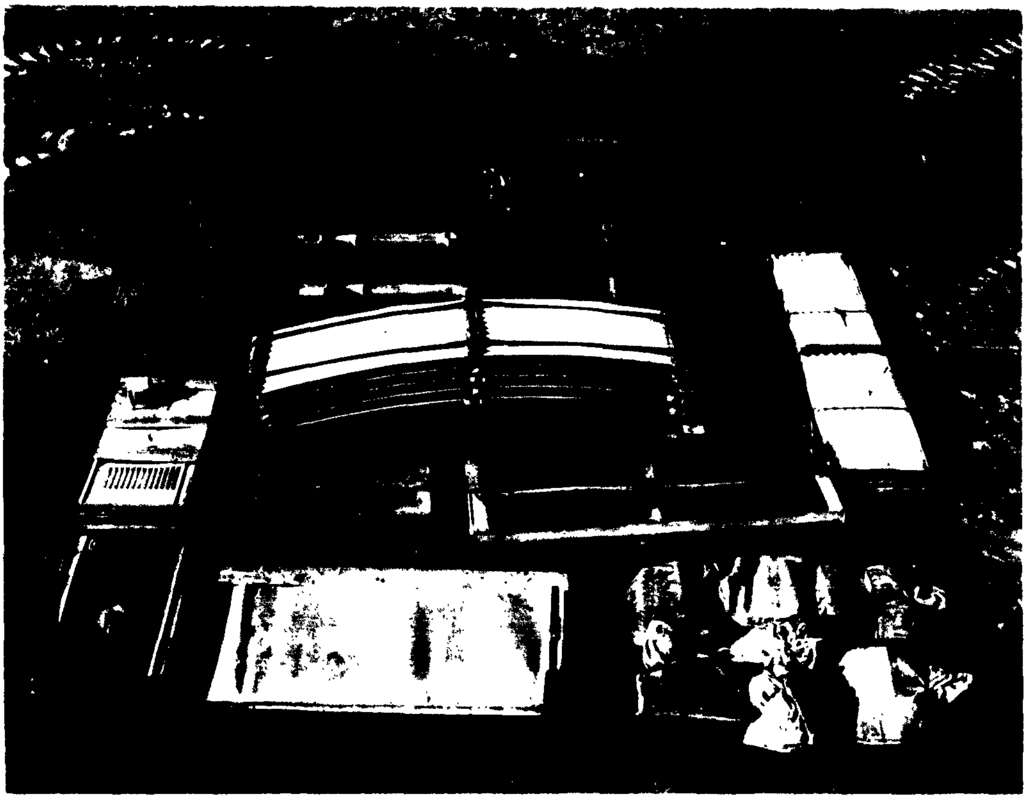


Figure 19. Building packaging.

Figure 20. Detail showing corrosion of purlins.

Table VII. Building Comparison Data

Building Description (all 40 by 100)	Reference No.	Total Cubage (cubic foot)	Total Weight (pounds)	Erection Time (manhours)
BuDocks Standard Arch-Rib	4	386	21,853	398
Wonder Building	5	445	26,139	315
BuDocks Standard Rigid Frame	6	509.5	27,585	429
Trim-Steel	—	274	19,525	323

Structure

The arches provide a factor of safety greater than 1.8 for wind and snow loadings. The structure was not tested under combination loadings, such as snow plus 50 percent wind, because, if a linear load-deflection relationship is assumed, the effects of combination loadings can be obtained by superposition. However, superposition is not truly valid because the axial thrust, N_φ, is a function of the load w. As the arch deflects, N_φ becomes eccentric by a distance y to the unstrained position of the arch (Figure 21); thus, an additional moment of intensity, $N_\varphi y$, is created. The stress at a point on the arch which is subjected to bending and axial thrust is

$$f_s = \frac{N_\varphi}{A} \pm \frac{M_\varphi c}{I} \pm \frac{N_\varphi yc}{I} = \frac{N_\varphi}{A} \pm \frac{c}{I}(M_\varphi \pm N_\varphi y) \qquad (1)$$

where the terms in parentheses represent the total moment, M_t, acting on the cross section. In Table III the bending and axial stress are those due to M_t and N_φ. Table III shows that the axial stress was only 6 percent of the total compressive stress; therefore, for small elastic deflections the stress condition for combination loadings may be obtained by superposition. But of course, superposition will not be valid for determining the stress at points on the arch which are subjected to localized stresses due to bearing, spreading, or buckling.

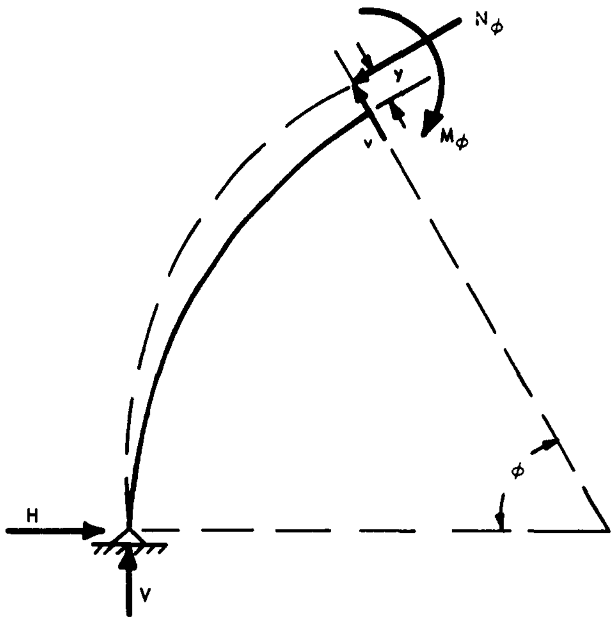


Figure 21. Sketch illustrating Equation (1).

The endwall framing withstood a 12.5-psf wind loading when subjected to axial compression introduced by a 20-psf simulated snow load. The strain in each stud over the double sliding door (Figure 15) was measured intermittently during test of the endwall framing, and the stress in these studs did not exceed 3,000 psi.

The stress in the edge leaf of the double sliding door varied linearly with load (gage S-5, Figure 15). Therefore, the stress of 26,370 psi (see Composite Framing Test) at 12 psf would give a proportional stress of 27,400 psi at 12.5 psf, which is the load specified in the test criteria. A stress of 27,000 psi is generally used as the allowable stress for members proportioned for wind load. The material exhibits a proportional limit of approximately 48,000 psi (Figure 22 in the Appendix), and the stress ratio of 48,000 to 27,000 is approximately 1.8.

When tested as a beam-column, the door jamb provided a factor of safety greater than 2.5. In Table VI the 300-pound loads are equivalent to that caused by a wind load of 16 psf on the double sliding doors. Since the combined load for design of the jamb would be expressed as snow plus 50 percent wind, the 16-psf wind load is greater than 2.5 times that specified for a 70-mph wind velocity.

Figure 18 indicates that the stiffness of the curved sheet was considerably greater than that of a flat sheet, and that the boundary condition at springing was neither fixed nor pinned. Criteria for design generally limit the allowable deflection for corrugated sheets to 1/90 of the span.[3] This limitation is imposed to prevent leaking at end laps or tearing at the bolt holes in end connections. On the arch structure the relative displacement between the ends of the sheeting is dependent upon the relative displacement of the purlins over which the sheeting spans. For the snow load condition, the relative displacement between the purlin at the crown and the two adjacent purlins was 0.55 inches at 180 percent of design load (from data for deflection gages D-5, D-6, and D-7, Figure 6a), and the relative displacement between the two adjacent purlins, the end supports for the sheeting, was only 0.15 inches due to a slight dissymmetry in loading. In accordance with the L/90 criteria,[3] the allowable relative displacement between these supports (Figure 18) of span L is 1.3 inches, and that of span L/2 is 0.65 inches, which is greater than the measured value of 0.55 inches. Therefore, the sheeting meets the criteria for deflection.

The ratio of the slopes for the experimental curve and the simple beam curve (Figure 18) is 9.8 which theoretically means that for a given deflection the arch should support a concentrated load 9.8 times greater than the load a simple beam would support when the beam stiffness, EI, is based on tabulated values.[3] A stress-load relation was not obtained for the sheeting because the L/90 deflection criteria should restrict the development of localized buckling in the corrugations and high stress concentration around the bolt holes.

Weathertightness

With the exception of the purlins, the protective finish of the structural members appears to be adequate. If the structure is dismantled and then re-erected, it may be difficult to obtain a watertight building. Approximately 5,000 field-drilled holes are required to stitch the lap joints in the sheeting; therefore, unless each sheet is piece-marked for re-erection, 5,000 additional holes will be required.

CONCLUSION

The Trim-Steel Building meets the minimum requirements of the criteria for Advanced Base Buildings.

FUTURE PLANS

The building, which is in use at the U. S. Naval Construction Battalion Center, Port Hueneme, California, will be studied to obtain information on corrosion and maintenance.

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NOMENCLATURE

A = cross-sectional area

b = intercept in equation $y = mx + b$

C = compression cell

Ch = hydraulic channel

c = distance to extreme fiber

D = deflection gage

E = modulus of elasticity

f_s = combined stress

$f(\varphi)$ = function of φ

H = horizontal thrust or reaction

H_A = thrust at point A

I = moment of inertia

K = radius of gyration

L = linear dimension

M_t = total moment

M_φ = moment as a $f(\varphi)$

m = slope

N. A. = neutral axis

N_φ = axial thrust as a $f(\varphi)$

P = load in pounds

p = concentrated load as a f(w, cosφ)

q = intensity of wind pressure on a vertical surface

R = radius

S = strain gage or stress

T = tension link

V = vertical reaction or wind velocity

v = radial shear

w = uniformly distributed load

x = linear dimension

y = deflection

Δs = differential increment of stress

Δw = differential increment of load

θ = subtended angle

φ = variable angle = ϕ

℄ = centerline

c-c = center to center

⊗ = deflection gage point

⯐ = hydraulic jack load point

+ = tension, or outward radial deflection

- = compression, or inward radial deflection

⇅ = direction in which deflection or translation occurred

Appendix

TEST DATA FOR PACKAGING, ERECTION, AND STRUCTURAL TESTS

Table VIII. Summary of Packaged Cubes and Weights

No. of Bundles	Dimensions (inches)	Weight (pounds each)
1	171 x 18 x 12	950
1	169 x 18 x 12	980
1	121 x 27 x 6	940
1	126 x 27 x 17.5	6,370
7	24 x 12 x 6 1/	705
5	234.5 x 12.5 x 8	1,276
2	86 x 36 x 4	100
2	48 x 36 x 5	70
2	54 x 29 x 5	70
2	87.5 x 7 x 1.25	40
1	47 x 27 x 15	390
1	46 x 6 x 3.5	30
5	240 x 14 x 5.25	340
2	98 x 15 x 6.75	80
Total	274 cu ft	19,525

1/ Estimated dimensions for cube of sacks.

Table IX. Tools Required for Erection

Quantity	Description
2	"Scruguns" — Skill Model 170 or Black and Decker Model 572, or approved equivalent with positive clutch, reversing switch, and shank adapters
2	Electric Drills — 1/4-in. heavy-duty type
2	Sockets — 3/8-in. 6-point heavy-duty impact type with 3/8-in. square drive
24	Twist Drills — 3/16-in. high-speed stubby type with split point — 135 degrees
2	Claw Hammers
1	Carpenter's Hand Level — 28-in. minimum length
1	Carpenter's Framing Square
2	Chalk-Line Reels
4	Pin Punches — 5/16-in. x 5/32-in.
2	Taper Punches — 9/16-in. -diameter for aligning 9/16-in. holes
1	Blacksmith Sledge — 3-pound
2	Side-Cutting Pliers
2	Crescent Wrenches — 12-in.-long
2	Screwdrivers — 12-in.-long to fit 5/16-in. overhead bolts
2	Tin Snips
2	Mastic Guns — for bulk mastic
1	Extension Cord — 300-ft heavy-duty
2	Extension Cords — 100-ft heavy-duty
2	Siamese Y's or 2-Socket Outlets — for connecting drill and "Scrugun"
4	Guy Lines — 1/2-in. manila, 50-ft-long
2	Ratchet Wrenches — 1/2-in. drive
2	Extensions — 4-in. for ratchet wrench
2	Standard Sockets — to fit 5/16-in.-square nut

Table X. Strain Data for Simulated Snow Loading[1]

Strain Gage No.[2]	Percent Design Load[3]						
	40	60	80	100	140	160	180
1	-5	-11	-24	-25	-25	-20	-18
2	-10	-28	-39	-50	-70	-79	-88
3	+188	+274	+380	+492	+710	+830	+960
4	-175	-270	-390	-523	-764	-910	-1,074
5	+173	+252	+361	+480	+690	+801	+920
6	-162	-252	-365	-480	-700	-818	-940
7	+170	+245	+354	+475	+680	+790	+910
8	-160	-248	-365	-480	-708	-820	-952
9	+170	+239	+340	+460	+660	+849	+945
10	-160	-240	-374	-461	-693	-810	-930
11	—	—	—	deleted	—	—	—
12	-190	-279	-425	-537	-790	-924	-1,058
13	+132	+172	+271	+353	+510	+587	+650
14	-175	-260	-430	-513	-760	-888	-1,015
15	+164	+237	+305	+432	+615	+712	+815
16	-187	-292	-451	-540	-789	-922	-663
17	+134	+205	+350	+470	+673	+777	+882
18	-177	-296	-429	-501	-739	-863	-994
19	+183	+270	+352	+488	+701	+809	+922
20	-172	-291	-410	-490	-720	-846	-977
21	-139	-258	-324	-470	-651	-776	-889
22	+150	+202	+280	+350	+520	+604	+675
23	-140	-245	-317	-438	-637	-751	-856
24	+145	+200	+276	+352	+530	+623	+700
25	-148	-240	-300	-438	-615	-727	-821
26	+160	+220	+300	+380	+565	+660	+740
27	-132	-210	-266	-372	-538	-630	-715
28	+176	+251	+342	+432	+641	+749	+839
29	-116	-178	-213	-304	-446	-523	-589
30	+211	+302	+398	+501	+740	+865	+970
31	-111	-180	-220	-300	-434	-508	-570
32	+200	+280	+370	+471	+699	+814	+912
33	-152	-239	-274	-421	-593	-700	-785
34	+158	+200	+290	+355	+560	+660	+740
35	-111	-252	-308	-468	-655	-762	-862
36	+180	+226	+330	+410	+620	+722	+810
37	-170	-258	-302	-452	-678	-799	-900
38	+160	+191	+290	+370	+548	+639	+711
39	+140	+208	+280	+345	+500	+583	+670
40	-188	-296	-400	-499	-710	-838	-962
41	-10	-19	-20	-26	-35	-42	-48
42	-5	-18	-22	-47	-69	-85	-101
43	-144	-198	-256	-338	-420	-460	-480
44	+101	+150	+242	+320	+488	+677	+670
45	+10	+5	+9	+2	0	+5	+10
46	+120	+165	+250	+330	+486	+568	+654
47	+111	+160	+230	+347	+460	+526	+599
48	+3	0	—	—	—	—	—

1/ Strain data are in microinches/inch.
2/ For location of gages see Figure 6.
3/ Design load = 20 psf.
\+ Tension.
\- Compression.

Table XI. Deflection Data for Simulated Snow Loading[1]

Gage No.	Percent Design Load[2]						
	40	60	80	100	140	160	180
1	+0.22	+0.28	+0.40	+0.58	+0.90	+1.02	+1.18
2	+0.32	+0.38	+0.55	+0.80	+1.22	+1.42	+1.60
3	+0.18	—	+0.20	+0.38	+0.68	+0.75	+0.82
4	-0.10	-0.30	-0.38	-0.40	-0.42	-0.52	-0.66
5	-0.48	-0.68	-0.87	-1.02	-1.48	-1.75	-2.03
6	-0.45	-0.72	-0.95	-1.27	-1.89	-2.25	-2.58
7	-0.42	-0.58	-0.78	-1.08	-1.60	-1.90	-2.18
8	-0.18	-0.10	-0.20	-0.38	-0.65	-0.75	-0.85
9	+0.10	+0.32	+0.48	+0.35	+0.42	+0.52	+0.60
10	+0.25	+0.51	+0.62	+0.72	+1.00	+1.18	+1.31
11	+0.20	+0.40	+0.48	+0.56	+0.78	+0.92	+1.08
12	+0.05	+0.05	+0.09	+0.15	+0.25	+0.30	+0.32
13	0	-0.02	-0.02	0	+0.05	+0.05	+0.05
14	+0.32	+0.38	+0.56	+0.81	—	+1.42	+1.62
15	+0.28	+0.25	+0.40	+0.61	—	+1.10	+1.28
16	0.08↓	0.02↑	0.03↑	0.02↓	0.15↓	0.12↓	0.11↓
17	+0.28	—	+0.41	+0.62	+1.01	+1.18	+1.31
18	—	-0.18	-0.25	-0.30	-0.46	-0.48	-0.60
19	-0.08	-0.15	-0.22	-0.28	-0.40	-0.48	-0.52
20	0.18↑	0.48	0.52	—	0.72	0.88	1.00↑

1/ Deflection data are in inches.
2/ Design load = 20 psf.
\+ Outward radial deflection.
\- Inward radial deflection.
↓ Windward movement.
↑ Leeward movement.

Table XII. Strain Data for Simulated Wind Loading[1]

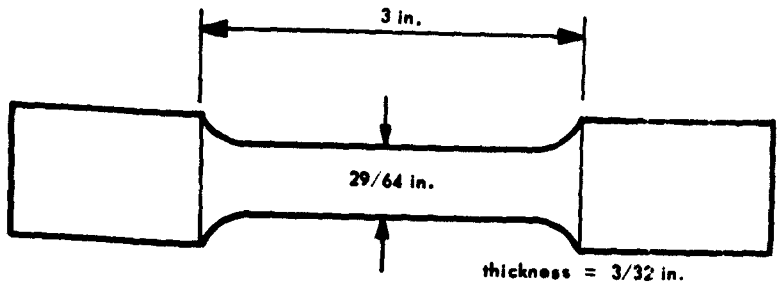
Strain Gage No.	Percent Design Load[2]				
	40	80	100	140	180
1	+25	+32	+102	+81	+78
2	+21	+25	+31	+42	+61
3	-158	-324	-406	-607	-806
4	+153	+301	+402	+603	+802
5	-151	-309	-405	-609	-800
6	+141	+281	+378	+568	+757
7	-149	-299	-390	-579	-752
8	+152	+308	+409	+610	+811
9	-168	-310	-389	-508	-650
10	+152	+311	+418	+621	+825
11	-100	—	—	—	—
12	+170	+338	+451	+680	+915
13	-118	-222	-290	-421	-549
14	+167	+331	+449	+670	+900
15	-112	-270	-345	-510	-666
16	+161	+322	+439	+658	+882
17	-227	-316	-405	-602	-788
18	+158	+303	+415	+621	+838
19	-158	-320	-421	-637	-839
20	+149	+286	+392	+585	+782
21	+50	+100	+93	+119	+180
22	-30	-97	-63	83	-111
23	+62	+111	+101	+130	+187
24	-42	-128	-125	-150	-162
25	+59	+113	+111	+149	+200
26	-41	-112	-91	-122	-154
27	+59	+109	+111	+150	+199
28	-40	-120	-100	-141	-180
29	+50	+91	+100	+138	+170
30	-61	-152	-142	-199	-243
31	+54	+102	+118	+148	+180
32	-64	-152	-149	-203	-243
33	+68	+130	+143	+191	+240
34	-56	-141	-132	-189	-221
35	+78	+147	+162	+220	+270
36	-57	-150	-140	-201	-238
37	+88	+162	+189	+256	+310
38	-52	-134	-133	-191	-220
39	+20	+51	+94	+112	+134
40	-13	-71	-112	-129	-133
41	0	+18	-21	+30	+51
42	—	—	—	—	—
43	+70	+79	+81	+109	+130
44	-112	-224	-296	-440	-588
45	—	—	—	—	—
46	-129	-279	-339	-492	-639
47	-109	-208	-270	-382	-500
48	0	-21	-53	-106	-147

1/ Strain data are in microinches/inch.
2/ Design load = 12.5 psf.
\+ Tension.
\- Compression.

Table XIII. Deflection Data for Simulated Wind Loading[1]

Gage No.	Percent Design Load[2]				
	40	80	100	140	180
1	-0.48	-0.88	-1.15	-1.60	-2.06
2	-0.60	-1.15	-1.60	-2.28	-2.92
3	-0.58	-1.08	-1.52	-2.18	-2.81
4	-0.48	-0.65	-1.00	-1.42	-1.88
5	-0.02	—	-0.22	-0.42	-0.48
6	+0.25	-0.50	+0.60	+0.83	+1.10
7	+0.45	+0.88	+1.18	+1.72	+2.28
8	+0.52	+1.02	+1.48	+2.18	+2.85
9	+0.50	+0.95	+1.42	+2.02	+2.68
10	+0.37	+0.72	+1.10	+1.58	+2.00
11	+0.20	+0.35	+0.52	+0.78	+1.01
12	-0.12	-0.22	-0.32	-0.42	-0.58
13	-0.18	-0.30	-0.46	-0.52	-0.82
14	-0.62	-1.21	-1.68	-2.40	-3.08
15	-0.68	-1.31	-1.82	-2.62	-3.40
16	0.58↑	1.10	1.50	2.12	2.75
17	-0.60	-1.15	-1.48	-2.28	-2.92
18	+0.10	+0.22	-0.25	-0.38	+0.50
19	+0.11	+0.25	+0.30	+0.40	+0.52
20	0.48↑	0.92	1.40	1.98	2.58

1/ Deflection data are in inches.
2/ Design load = 12.5 psf.
\+ Outward radial deflection.
\- Inward radial deflection.
↑ Leeward movement.



Tensile Specimen

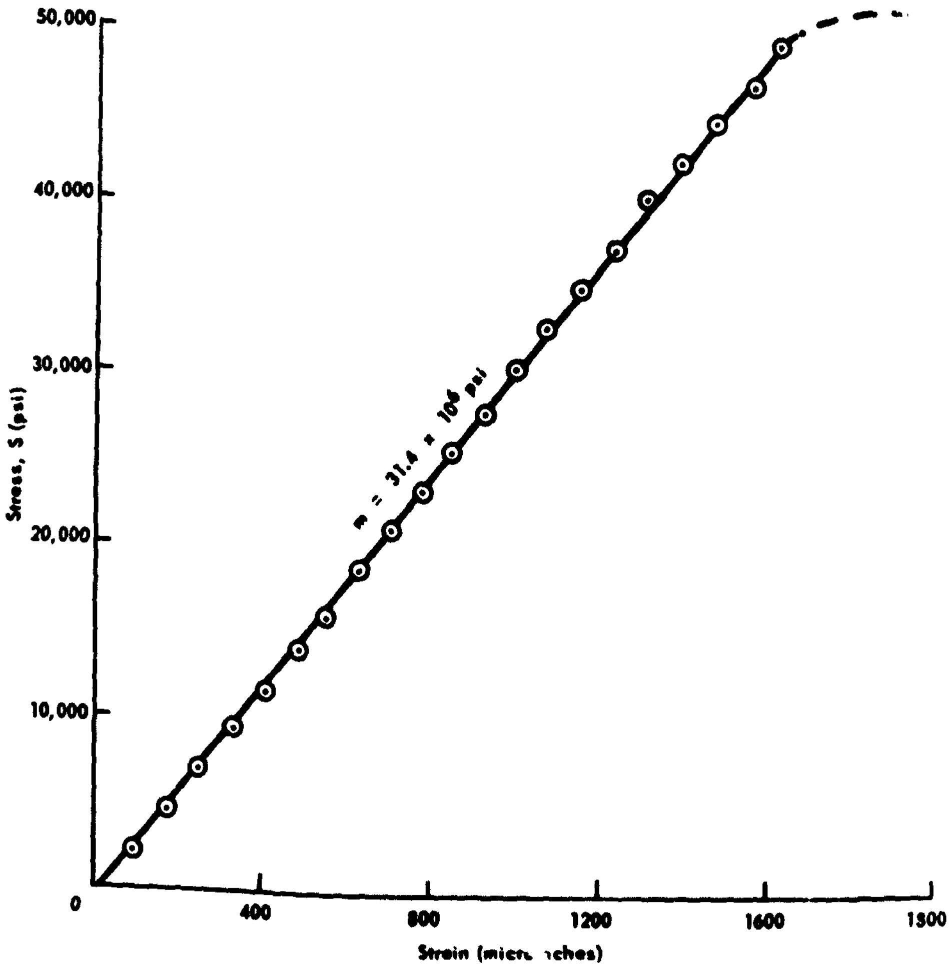


Figure 22. Stress-strain diagram of specimen from arch rib.

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39 p. illus 3 May 63 FOR OFFICIAL USE ONLY

A Trim-Steel Building was evaluated in accordance with the unfirom military requirements criteria for Advanced Base Buildings. The building meets the minimum military requirements.

1. Building — Evaluation
I. Webb, R. M.
II. Y-F015-99-029

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